INTERNATIONAL JOURNAL FOR NUMERICAL AND ANALYTICAL METHODS IN GEOMECHANICS
Int. J. Numer. Anal. Meth. Geomech., **23**, 1407–1425 (1999)

BACK ANALYSIS OF THE MEASUREMENTS PERFORMED DURING THE EXCAVATION OF A SHALLOW TUNNEL IN SAND

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SUMMARY

The results of back analyses based on the displacements measured during the construction of a railroad tunnel in an alluvial sand deposit are presented. After illustrating the characteristics of the construction technique, the main aspects of the *in situ* measurement programme and of the finite element approach used for back analysis are summarized. Then, the results of a series of calculations are discussed that concern both the determination of the 'secant' elastic moduli of the soil deposit and the attempt to identify the 'mechanism' governing the development of the surface settlements. On the basis of these results some comments are presented on possible improvements of the adopted excavation/construction procedure.

KEY WORDS: back analysis; cohesionless soil; jet grouting; finite elements; shallow tunnels; strain softening

1. INTRODUCTION

In situ measurements[1] are nowadays frequently carried out during the excavation of tunnels[2] in order to monitor the stability of the opening and, in the case of shallow tunnels, to control the effects of the induced surface settlements on pre-existing structures and buildings.

These measurements can also be used as input data in back analysis processes[3] to evaluate some relevant quantities (like e.g. the average mechanical properties of the *in situ* soil or the loads acting on the lining), or to refine their values assumed at the design stage.

In this context the back analysis consists in finding the values of parameters characterizing the soil mass that, when introduced in the stress analysis of the problem under examination, lead to results (e.g. displacements, stresses, etc.) as close as possible to the available *in situ* measurements.

In general terms, to perform a back analysis it is necessary to use two main 'tools'. The first one is a stress analysis procedure for determining the stress, strain and displacement distributions for the problem at hand. The second one is a suitable minimization algorithm which minimizes, with respect to the 'unknown' parameters of the soil mass, a non-linear function representing the discrepancy between the quantities measured in the field and the corresponding data obtained by the stress analysis.

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CCC 0363–9061/99/131407–19$17.50

Received March 1996
Revised 3 October 1997

Here an example of back analysis is presented which is based on the displacements measured during the construction of Monteolimpino 2 tunnel on the railroad connecting Milan (Italy) to Chiasso (Switzerland). This tunnel is excavated at a relatively shallow depth in an alluvial sand deposit and represents one of the early applications of sub-horizontal columns of jet-grouting as a support of the excavation face.

First, the general characteristics of the tunnel and of the construction technique adopted for driving it are illustrated. Subsequently, the main aspects of the *in situ* measurement programme and of the finite element approach used for back analysis are summarized.

Finally, the results of a series of calculations are discussed which have two main purposes. First, they aim at providing the 'average' values of the secant elastic moduli of the soil surrounding the tunnel. The comparison between them and those determined by *in situ* tests prior to the excavation permits to evaluate the effectiveness of the preliminary site investigation.

In addition, on the basis of the numerical results an attempt is made to identify the 'mechanism' taking place around the tunnel and that governs the development of the surface settlements.

Having reached an acceptable understanding of this mechanics, some comments are presented on the adopted excavation/construction technique in view of possible modifications and improvements of this procedure.

2. GENERAL CHARACTERISTICS OF THE TUNNEL

The construction of 'Monteolimpino 2' tunnel was initiated at the end of 1985 to improve the efficiency of the Gotthard railroad line connecting Italy to Switzerland.

This two-track tunnel has an internal radius of 5 m and a total length of 7.2 km. From its South adit (close to Albate, Italy), the tunnel first crosses alluvial and glacial deposits, for a length of about 250 m. Then it enters a sandy-marl formation having a total length of about 1500 m. Subsequently, close to Grandate plain, the tunnel crosses an alluvial deposit consisting mainly of sand and silty sand. This portion of the tunnel has a length of 750 m, an average depth of 35 m and is located above the water table.

The tunnel then continues through 4500 m of sound rock (first consisting of conglomerate and subsequently of limestone) and finally, crosses a glacial deposit of about 200 m in length.

Here the discussion is focused on the problems encountered in driving the tunnel in the alluvial deposit of dry sand. Additional information, concerning also the technology used for excavating the remaining portions of the tunnel, can be found in Reference 4.

To limit the difficulties of driving a tunnel in a dry sand, jet-grouted columns were adopted for consolidating the soil before the excavation. This, in fact, represents one of the early application of jet grouting for the excavation of tunnels.

With respect to the traditional 'permeation' grouting, jet grouting offers some advantages when used in fine grained soils, like silty sand. In fact, it eliminates the problems of penetrability, related to the small size of the pores, and strongly reduces the possible pollution caused by the chemicals used for permeation.

Jet grouting consists in the simultaneous fracturing and mixing of the soil *in situ* with cement grout injected at high pressure, up to 50 MPa. First, a drill is executed using a string of rods which carries at the tip a drilling and jetting tool. The jetting phase takes place during the drawing up of the drill string by injecting the cement grout (and, in some cases, air) through small radial nozzles located at the tip of the string.

The mechanical characteristics and the diameter of the columns of consolidated soil depend on the grout composition, velocity and pressure of the fluids, number and diameter of the nozzles, speed of rotation and withdrawal, as well as on the characteristics of the natural soil. In granular material the column diameter usually ranges between 50 and 70 cm.

Consolidated soil arches can be formed during the excavation of a tunnel by partially overlapping adjacent columns executed along the perimeter of its face.

The technique adopted for excavating the portion of Monteolimpino 2 tunnel in the sand deposit can be summarized in the following six steps which correspond to the numbers from 1 to 6 in Figure 1.

(1) A consolidated soil arch is formed, ahead of the excavation face, consisting of 35 jet grouted columns with an outward slope of 11° with respect to the tunnel axis. The holes are 1 m long. The first 3 m are simply plugged treatment, an overlap of 1 m exists between two subsequent arches. Figure 2 shows a longitudinal section of the sequence of consolidated arches.
(2) The top section of the tunnel (crown) is excavated for a length of 6 m. During excavation, a temporary lining is set in place, which consists of 1 m spaced steel ribs in contact with the jet grouted columns and of 15 cm thick shotcrete.
(3) Sub-vertical jet grouted columns are constructed at the base of the consolidated arch. The permanent concrete lining of the crown is set in place. During the subsequent excavation phases (4, 5) the inclined jet grouted columns operate as a foundation of the permanent lining of the crown.
(4) Two vertical sheet piles, about 4·5 m deep, are driven at a distance of 2 m from the tunnel centreline. The sheet piles are interrupted every 9 m to permit the subsequent construction of the first segments of the lower part of the permanent lining. The soil between the sheet piles is excavated.

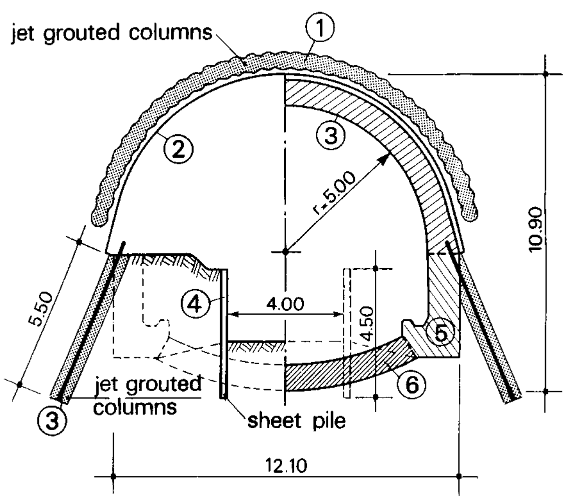


Figure 1. Steps 1–6 of the tunnel construction sequence in the sand deposit

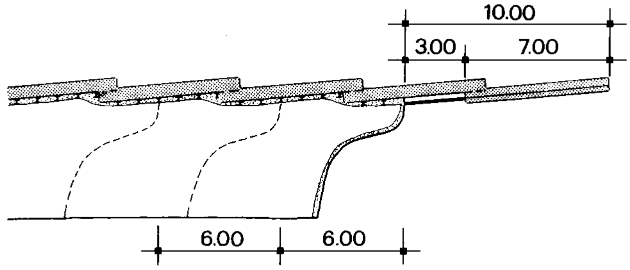


Figure 2. Longitudinal section of the tunnel showing the overlap existing between subsequent arches of sub-horizontal jet-grouted columns

(5) The lower part of the permanent lining is constructed by subsequent segments on alternate sides of the tunnel.
(6) The central sheet piles are removed; the lower part of the tunnel is excavated and the concrete inverted arch is completed.

3. SITE INVESTIGATION AND *IN SITU* MEASUREMENT PROGRAMME

Before initiating the construction of the tunnel a site investigation was carried out to quantitatively assess the geotechnical characteristics of the sand deposit. In particular, 50 m cored boreholes, standard penetration tests (SPT), and dilatometer tests,[5] for evaluating the *in situ* elastic moduli, were performed.

In addition, even though no major buildings were present in the Grandate plain above the tunnel, the surface settlements were controlled at two sections during the tunnel excavation through topographic survey. For each section the vertical displacements within the sand deposit along two vertical lines were also measured through sliding micrometers.[1] The first vertical line coincides with the tunnel centreline and the second one is located 10 m away from it.

The displacements recorded at the first instrumented section will be used in the following as input data of the back analyses.

Figure 3 shows the soil profiles for these sections. The dilatometer tests carried out at the first section indicated that, from the mechanical view point, the deposit can be roughly subdivided into three main layers. The first one from the surface to 10 m depth, the second one from 10 to 25 m depth and a third one which includes the remaining part of the deposit. Note that the values of the dilatometer modulus reported in Figure 3 correspond to the loading part of the tests.

The average values of the elastic moduli obtained for each layer from the loading (E_l) and unloading (E_u) stages of the dilatometer tests (cf. Figure 4) are reported in Table I.

As to the shear strength parameters, the maximum value of the peak friction angle obtained by the standard penetration tests was approximately $\phi = 40°$.

Figure 5 shows the locations where the vertical displacements were measured during the tunnel construction. The surface settlements measured at various stages of the advancing of excavation are shown in Figure 6. A maximum settlement of about 3 cm was observed at the end of the excavation of the top part of the tunnel. This settlement almost doubles after the completion of the opening.

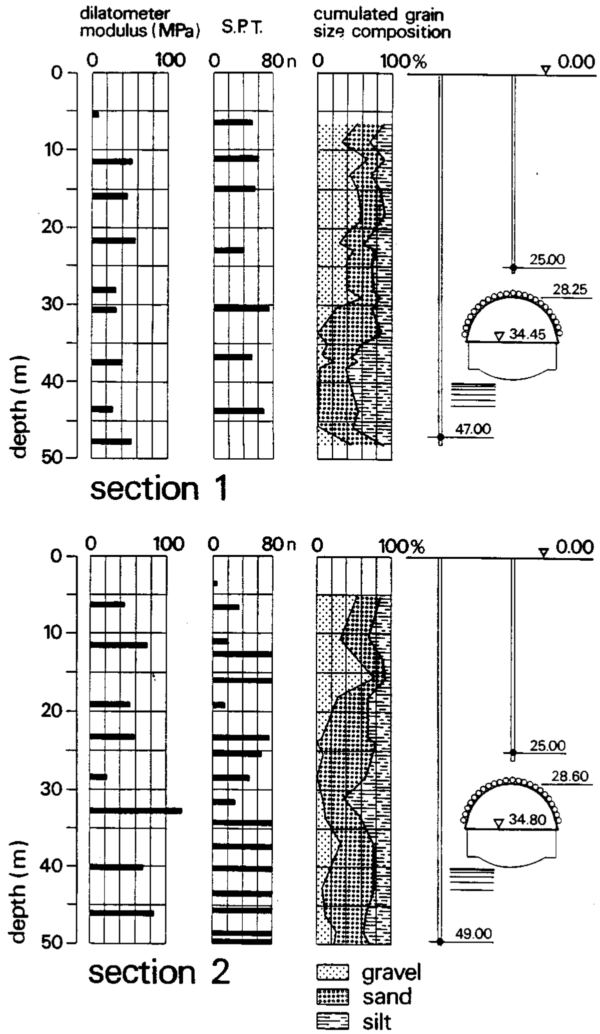


Figure 3. Soil profiles at Sections 1 and 2

As previously observed, the subsequent back analyses are based only on the data recorded in Section 1, where the maximum settlements took place. A more detailed description of the measurement programme and a complete report of its results for both instrumented sections can be found in Reference 4.

4. CHARACTERISTICS OF THE BACK ANALYSIS

In the geotechnical engineering context a back analysis consists in finding the values of the mechanical parameters, or of other quantities characterizing a soil or rock mass, that when

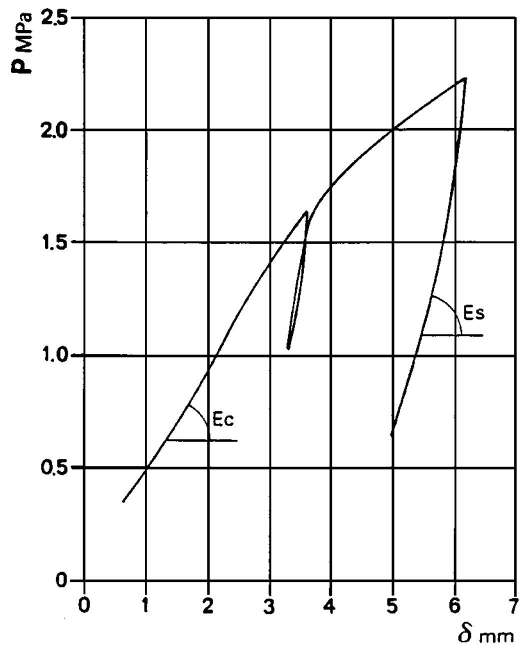


Figure 4. Pressure P vs. lateral displacement δ from a dilatometer test in sand

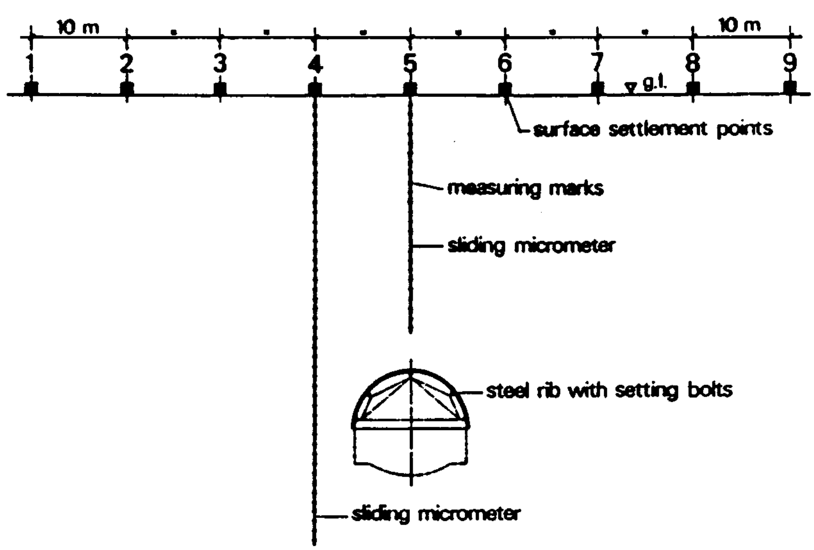


Figure 5. Locations of the surface points (1–9) where the vertical displacements of the two sliding micrometers are measured

Table I. Loading E_l and unloading E_u moduli of elasticity from *in situ* dilatometer tests

Layer no	depth (m)	E_l (MPa)	E_u (MPa)
(1)	0–10	10	140
(2)	10–25	45	320
(3)	25	30	260

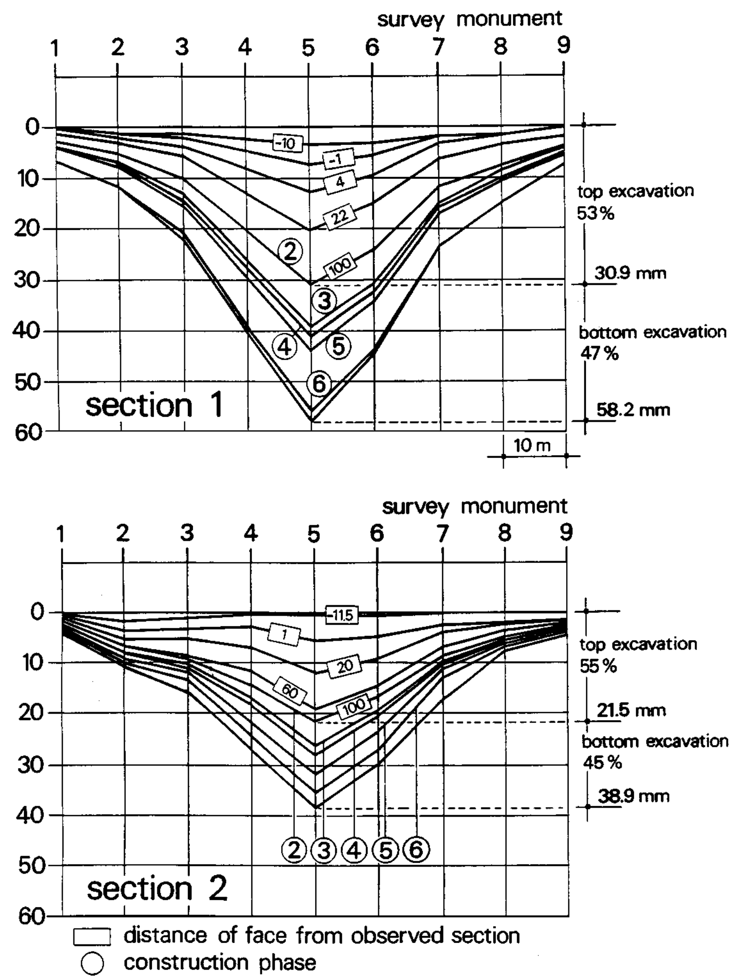


Figure 6. Surface settlements measured at Sections 1 and 2 during excavation. The circled numbers in Figure 6 correspond to the various stages of excavation depicted in Figure 1

introduced in the stress analysis of the problem under examination lead to results (e.g. displacements, stresses, etc.) as close as possible to the corresponding *in situ* measurements.

Various techniques have been proposed in the literature for solving this class of problems (see e.g. References 6–9). They are, in general, based on the minimization of an error function

representing the discrepancy between the field measurements and the corresponding quantities evaluated through a numerical stress analysis.

The error function depends, through the numerical results, on the mechanical parameters to be back calculated (which can have a rather general meaning and may correspond to elasticity or shear strength properties, viscosity coefficients, etc.). Consequently, the back analysis reduces to determining the set of parameters that minimizes the error function, i.e. that leads to the 'best approximation' of the field observation through the chosen numerical model.

The error is, in general, a complicated non-linear function of the unknown quantities, and in most cases, the analytical expression of its gradient cannot be determined. This is particularly the case for non-linear or elasto-plastic problems. Therefore, the adopted minimization algorithm must handle general non-linear functions and it should not require the analytical evaluation of their gradient.

Methods of this kind, known in mathematical programming as direct search methods,[10] are iterative procedures which perform the minimization process by comparing the values of the error function obtained in a sequence of successive evaluations of it. In the present context, each evaluation requires a finite element analysis of the tunnel excavation process based on the trial values of the mechanical parameters for their iteration.

For the purposes of this study the minimization process has been based on the following straightforward definition of the error function F,

$$F = \sum_{li}^{m} [u_i^* - u_i(\mathbf{p})]^2$$

Here, u_i^* are the m displacements measured in the field and u_i represent the corresponding numerical results which depend on the unknown mechanical parameters collected in vector $\mathbf{p}$.

In most practical cases, some limiting values exist for the unknown parameters. For instance, the modulus of elasticity cannot reach negative values. These limits, expressed by inequality constraints, can be easily taken into account in a minimization process based on direct search algorithm. In fact, if a point in the space of the free variables is reached outside the feasible domain, a very large value is assigned to the error function so that the minimization algorithm automatically drives back the optimization path into the feasible region.

5. RESULTS OF THE ELASTIC BACK ANALYSES

After the tunnel completion a back analysis based on the available *in situ* measurements was attempted in order to identify the 'mechanism' that governs the development of the displacements around the opening.

As previously observed, each iteration of this analysis requires a finite element simulation of the driving process of the tunnel. Even though this process has an essential three-dimensional nature, here the finite element calculations have been based on a two-dimensional, plane strain discretization. It will be shown, however, that in spite of this simplification introduced merely to limit the computational burden, the numerical results still provide some useful information on the previously mentioned 'mechanism'.

A first back analysis was carried out subdividing the sand deposit into six zones or layers, assuming for each of them a linear elastic behaviour and considering the corresponding Young

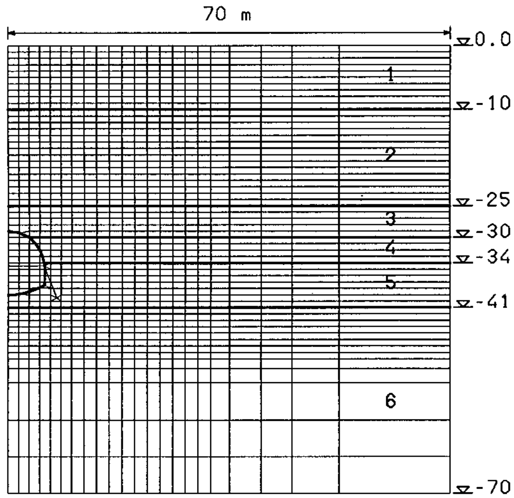


Figure 7. Finite element mesh subdivided into six soil layers

moduli as the free variables of the problem. Poisson ratio was kept constant for all layers and equal to 0·3.

The mesh adopted in the calculations is shown in Fig. 7. The soil and the grouted zone are discretized into 4 nods, quadrilateral elements, while one-dimensional beam elements are used for temporary and permanent supports.

In this analysis only the excavation of the top part of the tunnel is considered, which is referred to in the following as phase A of the excavation. The remaining part of the excavation, that includes the excavation of the bottom part of the tunnel, the installation of the jet grouting columns representing the foundation of the crown support and the completion of the permanent lining, is referred to as phase B.

Phase A of the tunnelling process was numerically simulated in two steps: first the elements discretizing the top portion of the tunnel are eliminated from the mesh and a set of forces equivalent to 30 per cent of initial stress distribution is applied to the opening contour. Then, the elements discretizing the crown support are introduced and the remaining part of equivalent forces is applied.

The initial stress state was estimated on the basis of the standard expression of the coefficient of earth pressure at rest $k_0 = 1 - \sin\phi$, which was considered realistic for the sand deposit under examination.

The choice of the percentage of the 'excavation forces' to be applied before introducing the crown support was based on a preliminary elastic–perfectly plastic analysis in which these forces were gradually increased without introducing any support. The results of this calculation showed that a rapid increase of the inward displacements of the opening contour takes place, in particular, in the zone between the springline and the bottom of excavation, when the applied forces exceeds about 30 per cent of their total values.

Since these relatively large displacements were not observed during construction, due to the presence of the arch of jet grouted columns before of the excavation face, it was decided to apply only 30 per cent of the total excavation forces during the first part of Phase A.

The vertical displacements corresponding to the 'optimal' values of the back calculated elastic constants at the end of phase A are shown in Figure 8 and compared with the *in situ* measurements. The diagram (a)–(c) represent, respectively, the vertical measurements along two vertical lines and the surface settlements.

It can be observed that a non-negligible difference exists between measured and calculated displacements along the vertical line (a) through the tunnel centre. In particular, the calculated vertical displacements (solid line) have an almost linear variation with depth while the measured displacements (dashed line) show a sharp increase close to the tunnel crown.

A possible explanation of this difference can be provided by the experimental data presented in Reference 11, which are derived from *in situ* measurements performed during the excavation of a subway tunnel. These data, reported for convenience in Figure 9, show that in the tunnel vicinity the shear strains tend to concentrate within two 'ear-shaped' zones initiating at the tunnel springlines.

A similar effect was also pointed out in Reference 12 where a series of finite element calculations were discussed concerning tunnels driven in cohesionless soil under various k_0 conditions. The numerical results obtained when the coefficient of earth pressure at rest is larger than 1 indicate that a plastic zone with a 'dome' shape develops close to the tunnel crown. On the contrary, if k_0 is smaller than 1 like in the present case, the plastic zone initiates close to the tunnel shoulders and develops upwards reaching a shape smaller to that reported in Figure 9.

A concentration of strains, like the one shown in Figure 9, might be associated to a reduction of stiffness, and shear strength, of the *in situ* soil. To check whether this effect has a relevant influence

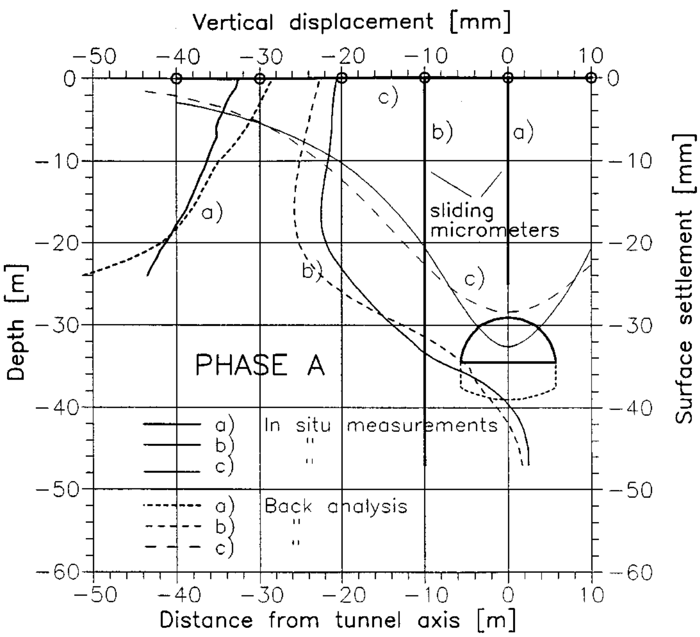


Figure 8. Comparison between the measured displacements (solid lines) and those obtained on the basis of the mesh in Figure 7 by the elastic back analysis (dashed lines) of the phase A of excavation. (a) Vertical displacements of the ground surface; (b) vertical displacements along the micrometer located at tunnel centerline; (c) vertical displacements along the micrometer located 10 m away from the tunnel centreline

for the problem at hand, it was decided to run a second elastic back analysis in which a loss of stiffness of the sand can takes place in a zone the geometry of which was *a priori* chosen.

In this case the finite element model is characterized by the narrow band indicated by a dark zone in Figure 10 the elastic properties of which may have values smaller than those of the sand layers. In particular, the elastic moduli in this zone are assumed equal to those of the corresponding layers multiplied by a reduction factor α constant for all layers. Therefore, the elastic moduli

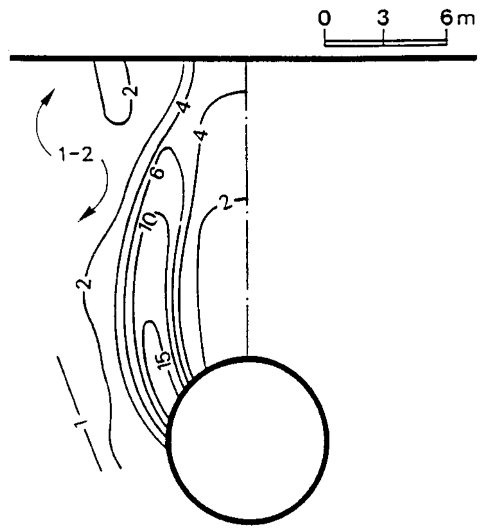


Figure 9. Contour lines of the maximum shear strain derived from the displacements measured during the excavation of a subway tunnel (after Reference 11). Contour line values are expressed in per cent

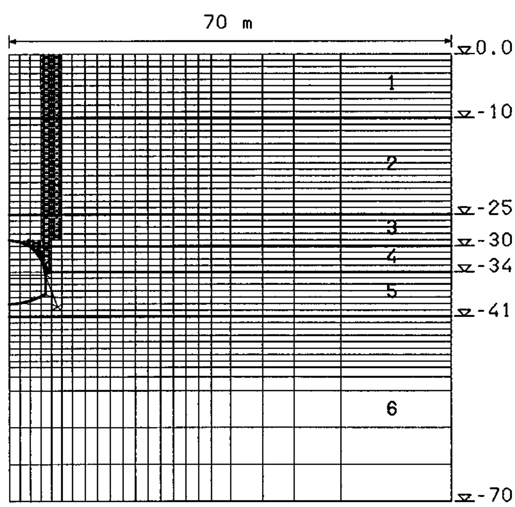
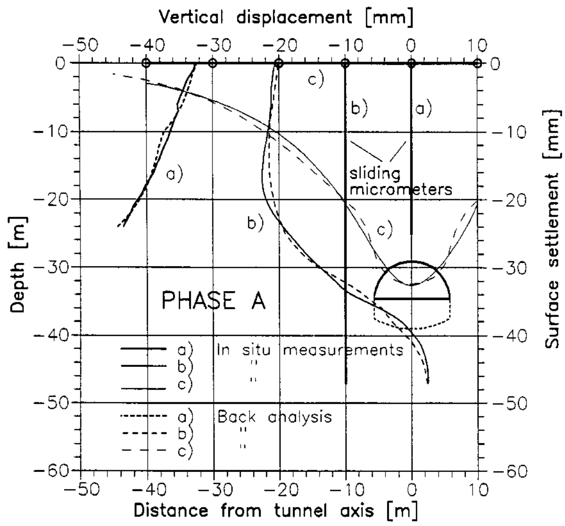


Figure 10. Modified finite element mesh containing a 'potentially softer' region (dark zone)

of the six soil layers and the reduction factor α represent the free variables of this second back analysis.

Two calculations were performed, which correspond to the excavation of the top part of the tunnel (phases A) and to the completion of the opening (phase B).

Figure 11 shows the comparison between the computed and measured vertical displacements at the end of the two phases. Note that due to the large settlement increase during the final step of excavation, two different displacement scales have been used for phases A and B.



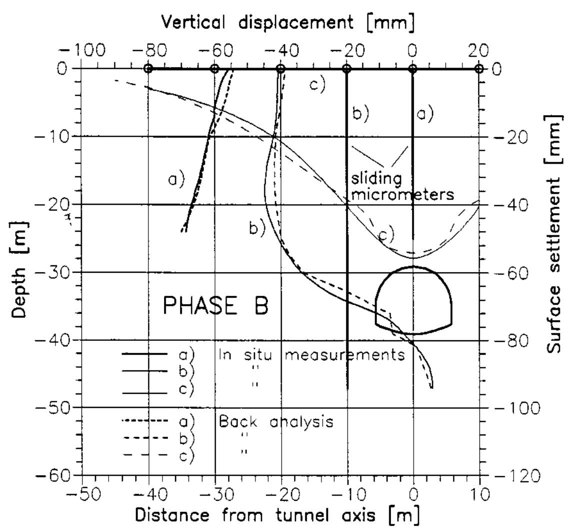


Figure 11. Comparison between the measured displacements (solid lines) and those obtained by the elastic back analyses (dashed lines) of the excavation phases A and B based on the modified finite element grid in Figure 10. Other characteristics as in Figure 8

Figure 12 presents a comparison between the loading and unloading moduli measured by *in situ* dilatometer tests and the elastic moduli obtained by the two back analyses. The reduction coefficient α was equal to 0·23 for phase A and to 0·26 for phase B. This indicates that the reduction factor has an almost constant values during the progress of excavation.

It can be also observed (cf. Figure 12) that the elastic moduli of the sand layers two and six obtained by the back analysis of the top excavation (phase A) are quite close to those calculated at the completion of excavation (phase B). On the contrary, the two back analyses led to substantially different moduli for the sand layer 3–5, the depth of which coincides with the tunnel depth.

This can be explained considering that the excavation phase B produces an appreciable incremental deformation of the soil close to the tunnel with respect to that obtained at the end of the top excavation (phase A). This incremental deformation is interpreted by the back analysis procedure as a reduction of the equivalent secant moduli of the sand layers in that zone.

These results could help in choosing the moduli of elasticity to be adopted in a preliminary linear analysis of a tunnel in sand, if loading and unloading moduli from *in situ* tests are available. Apparently, the modulus to be used for the zones in the vicinity of the tunnel should be close to the loading modulus. On the contrary, for the remaining part of the deposit, the average values between loading and unloading moduli seems preferable.

For completeness, in addition to the back analysis based on the mesh in Figure 10, another calculation was performed introducing a 'dome'-shaped zone affected by the parameter α, close to

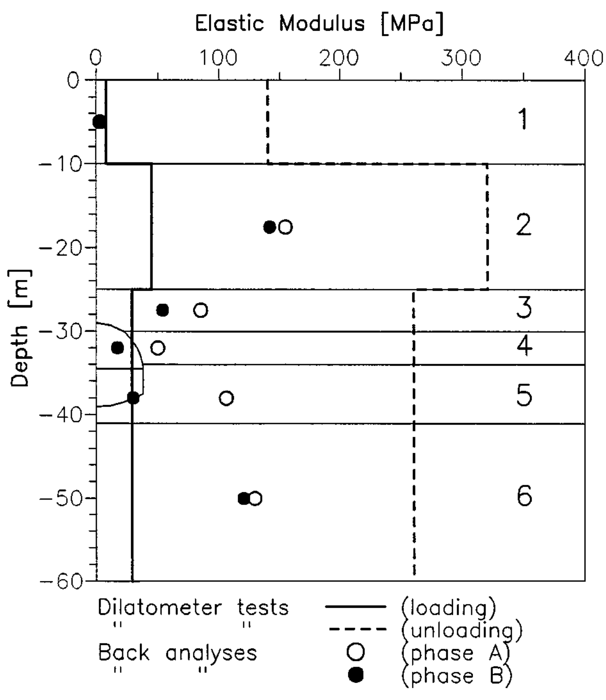


Figure 12. Elastic moduli of the six sand layers from the loading (solid line) and unloading (dashed line) stages of the dilatometer tests and from the back analyses of phases A (black dots) and B (circles) based on the modified mesh in Figure 10

the tunnel crown. This further analysis, however, did not lead to satisfactory results. In particular, the discrepancy between numerical and measured vertical displacements along line (a), already observed in Figure 8, did not change appreciably.

Even though the calculations described so far provide some insight into the deformation process around the tunnel they cannot reproduce the development of the plastic region in the sand deposit. To overcome this major limit it is necessary to remove the simplifying assumption of linear elastic behaviour of the sand.

6. ELASTOPLASTIC STRESS ANALYSIS

The elastoplastic stress analysis of the problem at hand was performed accounting for the loss of stiffness and strength of the sand surrounding the tunnel where plastic strains develop.[13]

This non-linear calculation is based on the same excavation phases already considered in the elastic back analyses. The elastic moduli are those obtained by the back analysis of the top excavation. The zone characterized by the reduction factor α is not introduced in the mesh.

The calculations are initiated assuming for all elements a 'peak' friction angle of 40°. The elements corresponding to the excavated sand are eliminated from the mesh and the nodal forces equivalent to their stress state are applied by small increments to the contour of the opening.

In the elements where the stresses reach the peak failure condition, the friction angle is reduced to its residual value, which is assumed equal to 30°. As a consequence, the current stress state in these elements exceeds the residual yield limit.

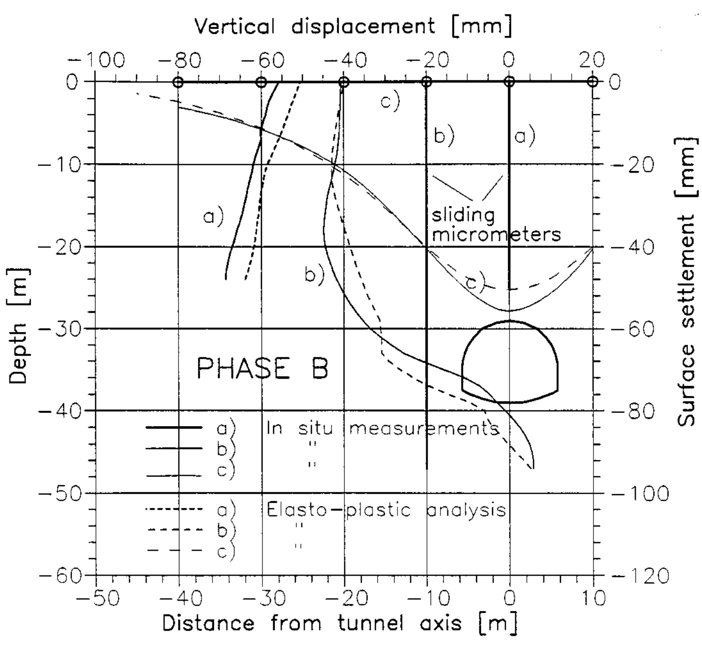


Figure 13. Comparison between the displacements measured at the completion of excavation (solid lines) and those obtained by the elastoplastic stress analysis (dashed lines) accounting for the loss of strength and stiffness of the sand. Other characteristics as in Figure 8

The 'unbalanced' nodal forces corresponding to the stresses in excess to the yield limit are evaluated by integration over the volume of the elements. Then, an iterative process based on modified Newton–Raphson method[14] is carried out to modify the stress distribution and to make it compatible with the current yield limit in all elements. During this process additional elements join the 'softened' region and a spreading of the plastic zone occurs in the sand surrounding the tunnel.

In addition to the friction angle, also the modulus of elasticity is reduced multiplying it by the average factor $\alpha = 0{\cdot}24$ obtained by the previous back analyses. During the solution, the nodal forces due to the difference between initial and residual moduli are added to those deriving from the loss of strength.

The iterations for the current increment of excavation forces continue until the number of involved elements stabilizes and the maximum unbalanced nodal force decreases below a chosen limit.

The results of this non-linear analysis are summarized in Figures 13 and 14. The final displacements at the end of excavation (phase B) are shown in Figure 13 and the corresponding

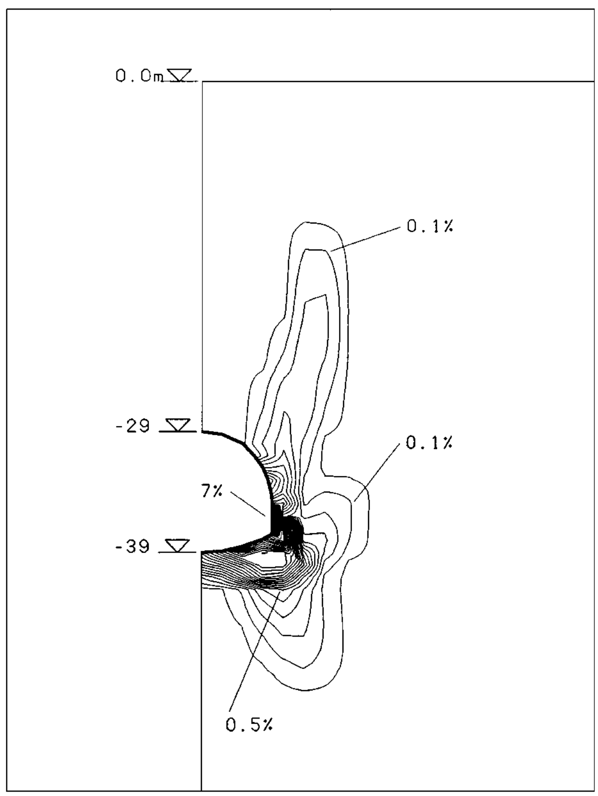


Figure 14. Contour lines of the square root of the second invariant of the deviatoric plastic strains from the elastoplastic stress analysis of the 'original' excavation procedure (min contour line value 0 per cent; max value 7 per cent; increment 0·1 per cent).

distribution of irreversible deformation, represented by the contour lines of the square root of the second invariant of the deviatoric plastic strains, is presented in Figure 14.

These results show that an elasto-plastic calculation, in which the loss of stiffness and shear strength is accounted for, is able to reproduce, at least from a qualitative view point, the strain concentration in the soil surrounding the tunnel experimentally observed in Reference 11 (cf. Figure 9). In addition, such a non-linear analysis provides an acceptable approximation (cf. Figure 13) of the displacements measured during construction.

Note that the elastic model, used in the first part of this study, was calibrated through a back analysis of the *in situ* measurements, while the elasto-plastic model was not 'turned' through a back analysis. Consequently, in this case the results of the elastic analysis provide a better approximation of the experimental data than those of the elasto-plastic one.

On the other hand, the elastic model cannot be used for the analysis of construction conditions different from the one previously described which, in general, could involve a different 'mechanism' governing the development of the displacements around the opening. This suggested to adopt the elasto-plastic model for the analysis of an improved construction procedure which is aimed at reducing the surface displacements induced by the excavation.

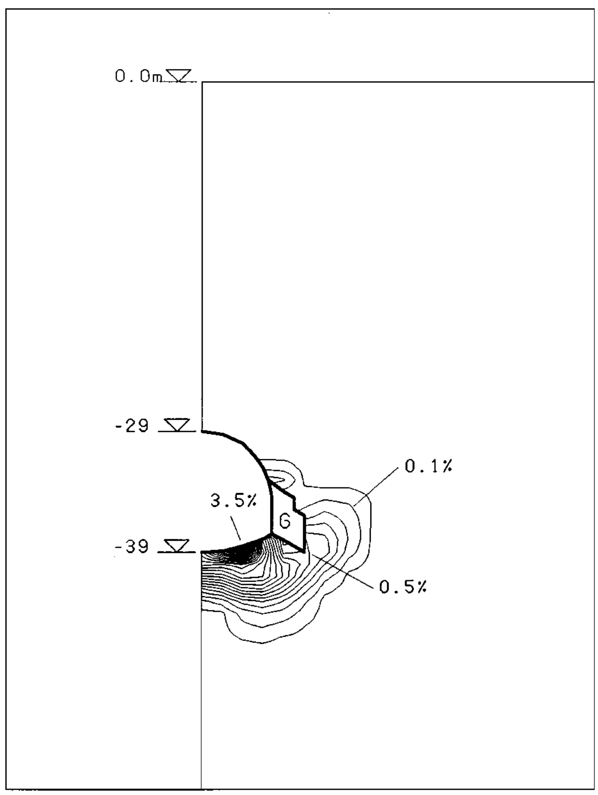


Figure 15. Contour lines of the square root of the second invariant of the deviatoric plastic strains from the elastoplastic stress analysis of the 'modified' excavation procedure. The additional jet grouting treatment is denoted by *G* (min contour line value 0 per cent; max value 3·5 per cent; increment 0·1 per cent)

7. ANALYSIS OF A MODIFIED EXCAVATION TECHNIQUE

The experimental data collected during construction indicate that an appreciable portion of the surface settlement takes place during the excavation of the bottom part of the tunnel (phase B). To reduce these settlements, in view of further applications of this excavation procedure, it was suggested to increase the bearing capacity of the foundation of the crown arch, for instance, by introducing additional sub-vertical jet grouted columns.

Since this provision involves technical problems and an increase of the cost of excavation, it would be desirable to check its effectiveness before adopting it in an actual project.

The non-linear finite element model discussed in the preceding section was considered as a s uitable tool for simulating the behaviour of Monteolimpino 2 tunnel in the presence of a modified construction procedure and for evaluating the consequent reduction of the surface settlements.

The change in the construction procedure is easily introduced in the finite element analysis simply by refining the mesh at the tunnel shoulders and assigning the property of the grouted soil to a larger number of elements with respect to those used in the previous calculations. The other characteristics of the analysis (e.g. the sequence of excavation/loading steps, the material model, the values of the material parameters, etc.) are unchanged.

Figure 15 shows the plastic zone at the end of excavation (phase B) where the contour lines of the square root of the second invariant of deviatoric plastic strains are plotted. In the same figure the zone '*G*' denotes the grouted zone that represents the modified foundation of the upper arch.

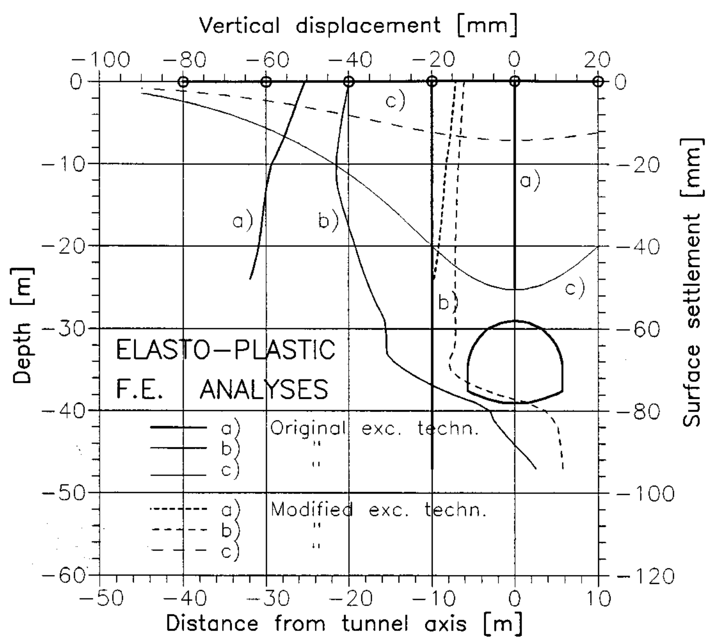


Figure 16. Comparison between the vertical displacements calculated on the bases of the original (solid lines) and modified (dashed lines) excavation techniques. Other characteristics as in Figure 8

It can be observed that the modified construction procedure involves a marked reduction of the extension of the plastic zone with respect to that characterizing the original procedure. This, in turn, leads to a decrease of the surface settlements, as shown in Figure 16 by the comparison of the vertical displacements calculated for the two excavation procedures.

As previously observed, the accuracy of the elasto-plastic model could be improved, since its parameters were not calibrated on the basis of a back analysis. In spite of this, some useful information is obtained by the numerical analysis, concerning, in particular, the surface settlements which are reduced by a factor of about 2–2.5 with respect to the original excavation procedure. This information can be used, together with those related to the increment of the cost of excavation, to the effects of the reduced (but not negligible) settlements on buildings in the construction area, etc. as one of the parameters for evaluating the effectiveness of the modified technique.

8. CONCLUDING REMARKS

The results of this study show that a back analysis may provide a useful insight into the overall behaviour of the soil mass surrounding a tunnel, even under the limiting assumption of linear stress–strain regime. In the present case the calculations led to a hypothesis concerning the formation of an 'ear-shaped' zone, initiating at the tunnel shoulders, where the strains tend to concentrate.

The linear elastic model, however, presents a major limit. In fact, due to the non-linear nature of the stress–strain relationship of the soil, it cannot predict the behaviour of the tunnel under conditions different from those in which the *in situ* measurements have been performed.

To overcome this drawback a simple elastoplastic model was adopted in which the modulus of elasticity and the friction angle of the sand were allowed to decrease with increasing permanent shear strains.

For the problem under examination, this finite element model was able to provide a reasonable approximation of the settlements and of the vertical displacements actually measured in the field.

This model was then applied to the analysis of an improved excavation technique. The results of calculations permit to estimate the reduction of the surface settlement, which can be used as one of the parameters for evaluating the effectiveness of the modified technique.

On these bases it seems possible to conclude that back analyses are able to provide a better understanding of the actual behaviour of tunnels during construction. They may also lead to numerical models which may help the designer in evaluating the effects of some limited changes, or improvements, of the construction procedure and in assessing the actual advantages of adopting the modified technique in subsequent applications.

ACKNOWLEDGEMENTS

The authors wish to thank A. Cividini for her valuable contribution and suggestions.

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